

DESIGN AND CONSTRUCTION BENEFITS OF PERFORMING A BI-DIRECTIONAL TEST ON A DRILLED SHAFT IN SAND, GRAVEL, AND COBBLE DEPOSITS

Van E. Komurka, P.E., D.GE, F.ASCE, GRL Engineers, Inc., Cleveland, Ohio, USA, 216.831.6131, vkomurka@grlengineers.com

Marco Fellin, P.E., Tetra Tech, Inc., Missoula, Montana, USA, 406.543.3045, marco.fellin@tetrattech.com

ABSTRACT

Significant cost and construction benefits were obtained from performing a bi-directional static load test prior to construction on the Russell Street Bridge Replacement project in Missoula, Montana. The project utilized 24 drilled shafts to support two three-span bridges across the Clark Fork River. A bi-directional static load test (“BDSLTL”) was performed on a large-diameter production shaft to demonstrate construction methods and confirm design assumptions. Subsurface conditions consisted of gravel with sand to a depth of 20 feet (6.1 m), underlain by gravel with sand, silt, cobbles, and occasional boulders. The test shaft had a nominal diameter of 5.9 feet (1.8 m), and an embedded length of 75.5 feet (23.0 m). The geotechnical engineer of record and the BDSLTL test engineer collaborated to optimize the BDSLTL jack assembly location within the test shaft, determine appropriate instrumentation locations, and plan the test protocol. The maximum applied test load was 10,707 kips (47.6 kN). Significant benefits were obtained from performing the BDSLTL test, including 1) reducing each production shaft’s length by using a higher LRFD resistance factor allowed by having performed the testing, 2) using test-determined unit shaft resistances to evaluate and accept two production shafts that may have otherwise required remediation, and 3) providing the Montana Department of Transportation (and other designers) with increased presumptive unit resistance values in the gravel/cobble/boulder deposits.

PROJECT DESCRIPTION

The project was constructed in Missoula, Montana for the Montana Department of Transportation (“MDT”). The original bridge was approximately 60 years old, and consisted of a two-lane, 420-foot-long (128-m), four-span, riveted steel plate girder design spanning the Clark Fork River. The bridge was due for replacement owing to its deteriorated state, and the need for increased traffic volume capability afforded by a four-lane structure. HDR was the structural engineer. The preferred bridge alternate was a 459-foot-long (140-m), three-span, welded steel plate girder design. The new bridge was designed to meet current standards in accordance with MDT bridge design and AASHTO LRFD criteria. The new bridge was constructed in phases to keep traffic open during construction.

Based on early design type-size-location meetings, drilled shafts were the preferred foundation type for both abutments and the river piers, primarily due to environmental concerns related to the endangered bull trout population. Tetra Tech, Inc. of Missoula, Montana was the geotechnical engineer of record. Geotechnically, drilled shafts were the preferred foundation type due to the ability to advance through the anticipated cobbles and boulders.

The specific design methodologies used for the bridge foundation design were 1) LRFD Bridge Design Specifications (AASHTO, 2012), 2) Guide Specifications for LRFD Seismic Bridge Design (AASHTO, 2011), 3) the FHWA’s drilled shaft design manual (FHWA, 2010), and 4) a Utah Department of Transportation (“UDOT”) publication (Rollins, 1997). Hydraulic information provided by HDR indicates that the 100-year flood level would cause up to 15.0 feet (4.57 m) of pier scour, and no contraction scour.

Drilled shaft axial nominal resistances were calculated using the computer program SHAFT 6.0 (Ensoft, 2007). The SHAFT 6.0 program contains a gravel layer nominal resistance estimate based on the results of a UDOT study (Rollins, 2007). The analysis assumes that the gravel layer has greater than 50 percent

gravel content (which is the case for this project). The UDOT study proved through static tension load testing that soils with greater than 50 percent gravel content have a nominal resistance on the order of 170 to 300 percent greater than predicted by the FHWA ‘sand’ equation (Brown, et al., 2010). The gravel layer formula used by SHAFT 6.0 typically produces the same 170 to 300 percent increase over the FHWA formula, while still accounting for the load testing data scatter.

The project was designed to include a large number of drilled shafts ranging in diameter from 4 to 6 feet (1.2 to 1.8 m). Based on potential overall cost savings, as well as on obtaining valuable soil behavior information, a bi-directional static load test was specified to be performed during construction on the first production shaft. This allowed for the use of an increased geotechnical resistance factor for design, reducing required shaft lengths based on axial demands. Based on the LRFD design manual (AASHTO, 2012), and discussions and concurrence with the MDT Geotechnical Section, the LRFD resistance factor associated with performing a BDSLT was 0.70. The resulting design totaled 26 drilled shafts: eight 5.9-foot-diameter (1.8-m) river shafts (each with an embedded length of 65 feet (20 m), and 18 4.0- to 4.5-foot-diameter (1.2- to 1.4-m) abutment shafts (ranging in embedded length from 30 to 35 feet (9.1 to 11 m)). Construction on the test drilled shaft and northbound bridge started in March 2018, and the southbound drilled shafts are scheduled to be completed Spring 2020. Total construction cost is estimated to be \$29M.

SUBSURFACE CONDITIONS

Four borings were drilled (one at each of the four bridge bent locations) during the geotechnical investigation, to depths on the order of 110 to 120 feet. In addition, samples were collected at 10-foot intervals during test shaft construction, and bulk samples were tested in the laboratory. In general, native poorly to well-graded gravel with sand, frequent cobbles, and occasional boulders were encountered during the geotechnical investigation and in the test shaft excavation. Occasional sand seams/layers, and gravel with higher percentages of sand, were encountered at all five explored locations. Native soils were predominantly subrounded to subangular quartzite, with a minor amount of argillite intermixed.

Subsurface Profile Used for Design

Figure 1 presents a generalized subsurface profile at the test shaft location based on conditions encountered in Boring BH-2 (located approximately 85 feet (26 m) from the test shaft), the estimated soil properties used for design, and for selecting the jack assembly location within the test shaft.

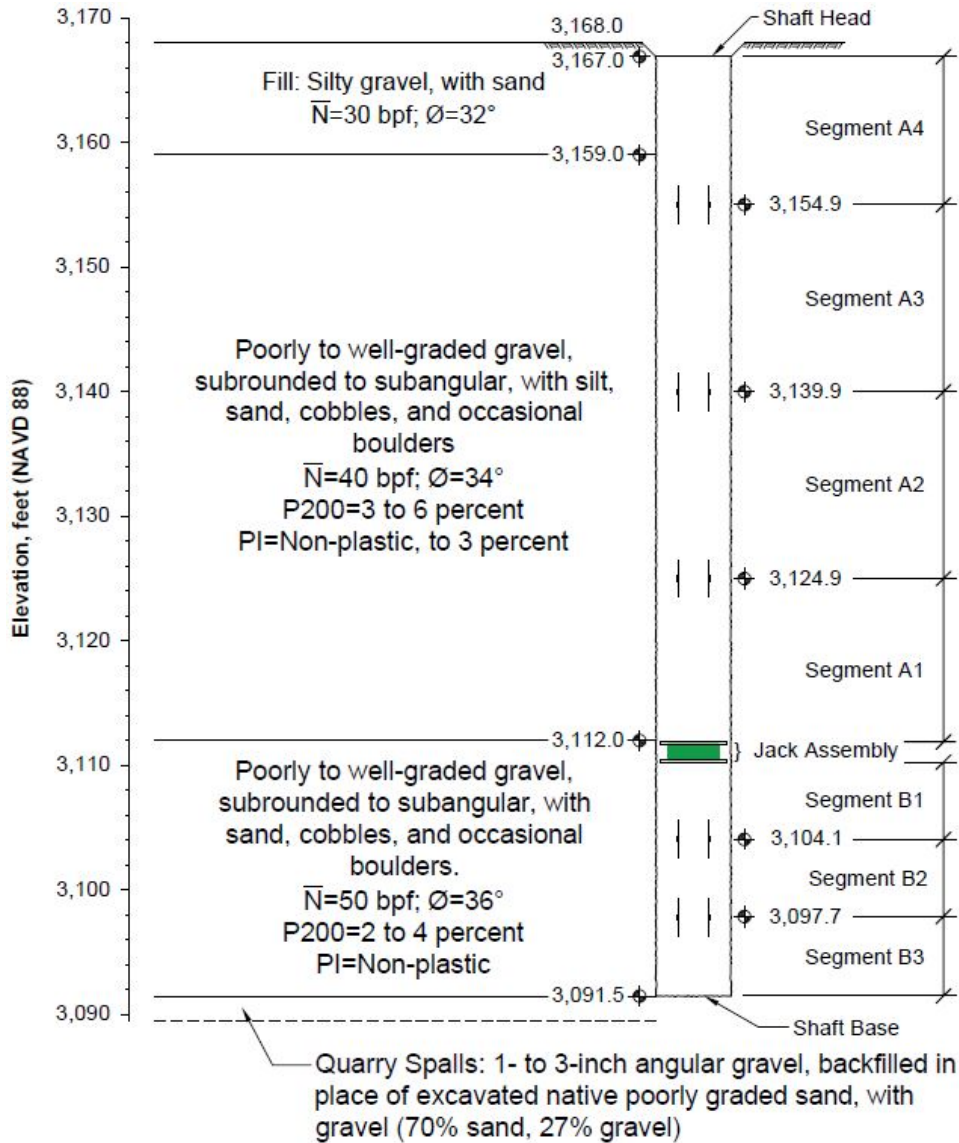
Subsurface Profile Encountered in Test Shaft Excavation

Table 1 presents laboratory testing results from test shaft excavation samples, which are similar to laboratory testing results from the geotechnical investigation. The table includes quarry spalls, which were placed in a zone two feet below the planned drilled shaft base to inhibit base softening following drilling.

TEST SHAFT DESIGN AND CONSTRUCTION

Design

Prior to the test, in an effort to optimize data obtained from the BDSLT, the jack assembly location (i.e., elevation) within the test shaft, and the minimum required test load and jack assembly capacity, were jointly decided upon. The jack assembly lower bearing plate was re-located to approximately 19 feet (5.8 m) above the shaft base (Fig. 1). Jack assembly capacity was increased to 6,600 kips (29,360 kN) with the use of three 2,200-kip (9,786 kN) **GRL-Cells** (Fig. 2), resulting in a maximum potential test load of 13,200 kips (58,716 kN) (6,600 kips (29,360 kN) upward, plus 6,600 kips (29,360 kN) downward). Instrumentation



“bpf” = blows per foot. 1 foot = 0.3048 m.

Fig. 1. Generalized subsurface and test drilled shaft profile.

included determination of jack assembly upper and lower bearing plate displacement, digital levels to monitor head displacement, base telltales, and five strain gage levels (Fig. 1).

Before the test, the estimated geotechnical nominal resistance above the jack assembly was approximately 125 percent of the geotechnical nominal resistance below the jack assembly: approximately 2,700 kips (12,010 kN) shaft resistance above the jack assembly, and 980 kips (4,360 kN) of shaft resistance and 1,180 kips (5,249 kN) of base resistance below the jack assembly, for a total estimated nominal resistance of 4,860 kips (21,620 kN). The jack assembly location was selected based on the assumption that as the base experienced displacement during the test, it would exhibit greater nominal resistance than estimated. The additional jack assembly capacity was intended to increase the likelihood of mobilizing all the resistance either above or below the jack assembly.

Table 1. Shaft Excavation Samples Laboratory Testing Results Summary

Depth, feet	Elevation, feet ^A	AASHTO Classification	ASTM Classification	P200 ^B	Plasticity Index, PI
0-8	0-3,159	A-1	Fill: Poorly-Graded Gravel, with Sand	Not Tested	Not Tested
8-30	3,159-3,137	A-1	Well-Graded Gravel, with Sand	3	NP
30-40	3,137-3,127	A-1	Poorly Graded Gravel, with Silt and Sand	6	3
40-50	3,127-3,117	A-1	Well-Graded Gravel, with Sand	3	NP
50-60	3,117-3,107	A-1	Well-Graded Gravel, with Silt and Sand	4	NP
60-70	3,107-3,097	A-1	Poorly Graded Gravel, with Sand	4	NP
70-72	3,097-3,095	A-1	Poorly Graded Gravel, with Sand	2	NP
72-74	3,095-3,093	A-1	Well-Graded Gravel, with Sand	2	NP
74-75.5	3,093-3,091.5	A-1	Poorly-Graded Gravel, with Sand	3	NP
75.5-77.5	3,091.5-3,089.5	A-2	Poorly-Graded Sand, with Gravel	3	NP
75.5-77.5 ^C	3,091.5-3,089.5	Quarry Spalls: 1- to 3-inch Gravel		0	NP

Notes:

A. NAVD 88.

B. "P200" denotes the percent (by weight) of material passing the #200 sieve size.

C. Native material was excavated, and quarry spalls were backfilled and compacted with 'flat-pan' on end of Kelly bar prior to reinforcing cage placement.

Construction

The test shaft was constructed in a production shaft location beneath a retaining wall. The retaining walls were founded on smaller-diameter drilled shafts than the river piers. To replicate river-shaft dimensions, the test shaft was constructed with the same diameter as the river shafts. However, to accommodate connection to the retaining wall it supported, the test shaft's reinforcing cage diameter was typical of a 4.5-foot-diameter (1.37-m) drilled shaft used to support the wall.

The project was awarded to Dick Anderson Construction of Missoula, Montana in February 2018, who subcontracted drilled shaft excavation to Inland Foundation Specialties of Boise, Idaho. GRL Engineers, Inc. of Cleveland, Ohio was contracted to perform the bi-directional static load test. The test shaft was the first production shaft constructed, was 70.9 inches (1,800 mm) in diameter, with an embedded length of 75.5 feet (23.0 m). Test shaft excavation was accomplished using a Casagrande B300SP drill rig with auger and cleanout bucket. Temporary casing was installed full-depth using a CMV casing oscillator connected to the drill rig. Fig. 3 illustrates typical drilling spoil.



Fig. 2. Jack assembly, showing upper and lower bearing plates, and three 2,200-kip GRL-Cells



Fig. 3. Typical subrounded to subangular poorly graded gravel, with sand, cobbles, and boulders (from 40 feet)

BI-DIRECTIONAL STATIC LOAD TEST RESULTS

The bi-directional static load test was performed on June 6th, 2018. Fifty-four test load increments of approximately 200 kips (890 kN) each were applied, with each load increment held for ten minutes, resulting in a maximum test load of 10,707 kips (47,627 kN). 200-kip (890-kN) test load increments were selected to provide a reasonable number of load increments based on the pre-test estimate of total nominal resistance. The large number of test load increments resulted from the mobilized nominal resistance exceeding pre-test estimates.

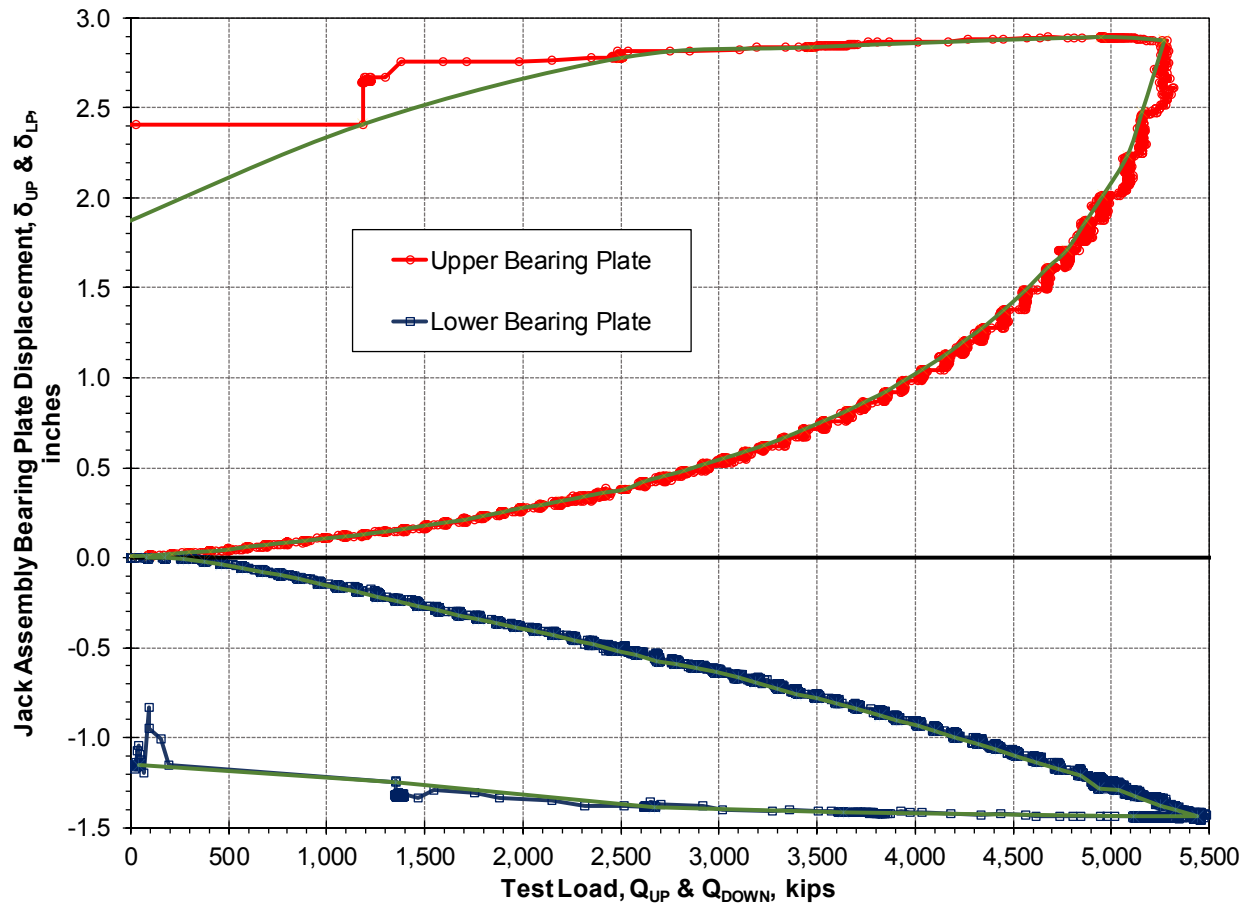
Bearing Plate Displacement

Upper and lower jack assembly bearing plate displacement as a function of test load is presented in Fig. 4. A review of Fig. 4 indicates that the test was limited by geotechnical “ascending” failure of the foundation portion above the jack assembly at an approximate upward test load of 5,270 kips (23,440 kN). Accordingly, shaft resistance was likely fully (or nearly fully) mobilized above the jack assembly, and likely not fully mobilized below the jack assembly.

Internal Force Profiles

Internal forces were known at three other locations in the test shaft: at the head (zero), and at the upper and lower bearing plates (upward and downward test loads, respectively). At each strain gage level, average measured strains were converted to internal force. Internal force at the base (equal to total base resistance, Q_{BASE}) was estimated by assuming that the lowest foundation segment (Segment B-3, Fig. 1) experiences the same average unit shaft resistance as the segment above it (Segment B-2, Fig. 1). The calculated internal force profiles are presented in Fig. 5. For clarity, results from every other load increment are plotted.

The average unit shaft resistance versus segmental midpoint displacement was calculated for each foundation segment. For each applied test load increment, the internal force profiles were used to calculate the average unit shaft resistance, t , for each segment by dividing the difference in boundary forces by the foundation segment’s shaft surface area. Head displacement was measured using digital levels. Base displacement, z_{BASE} , was determined using base telltales. Segmental elastic compression was calculated using strain data. Segmental displacements, z , were calculated by summing elastic compression downward



1 inch = 25.4 mm; 1 kip = 4.448 kN

Fig. 4. Jack assembly upper and lower bearing plate displacement

from the head above the jack assembly, and upward from the base below the jack assembly. Unit base resistance, q_{BASE} , was calculated by dividing total base resistance by the base area.

The relationship between shaft displacement and unit shaft resistance is presented in Fig. 6. The relationship between base displacement and base resistance is presented in Fig. 7. The corresponding equivalent top-loading (“ETL”) curves generated from the $t-z$ and $Q-z_{BASE}$ relationships, with differentiated shaft and base loads, are presented in Fig. 8.

A review of Fig. 1 indicates that for analysis, Segment A1’s lower bound was established at the top of the jack assembly upper bearing plate. Establishing Segment A1’s lower bound at the top of the jack assembly lower bearing plate decreases Segment A1’s unit shaft resistance values presented in Fig. 6 by 10.5 percent.

BENEFITS OF BI-DIRECTIONAL STATIC LOAD TESTING

Increased Resistance Factor

As discussed previously, per LRFD design methodologies, a higher resistance factor can be used if top-loading or bi-directional static load testing is performed. Without static load testing, the LRFD resistance factors for drilled shafts in sand are 0.55 and 0.50 for shaft and base resistance, respectively. Because a BDSLT was performed, a resistance factor of 0.70 was used for this project.

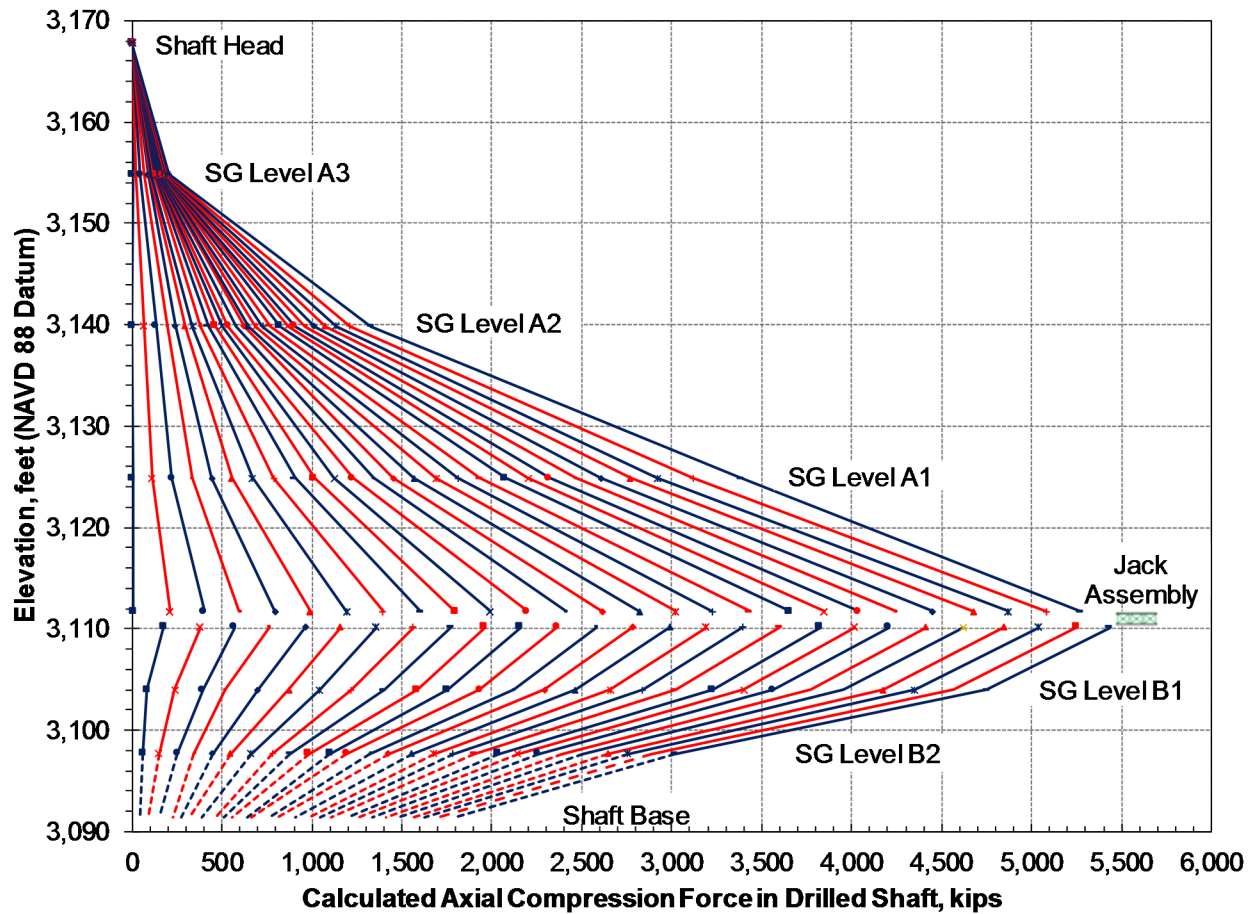


Fig. 5. Calculated Axial Internal Compression Force Profiles

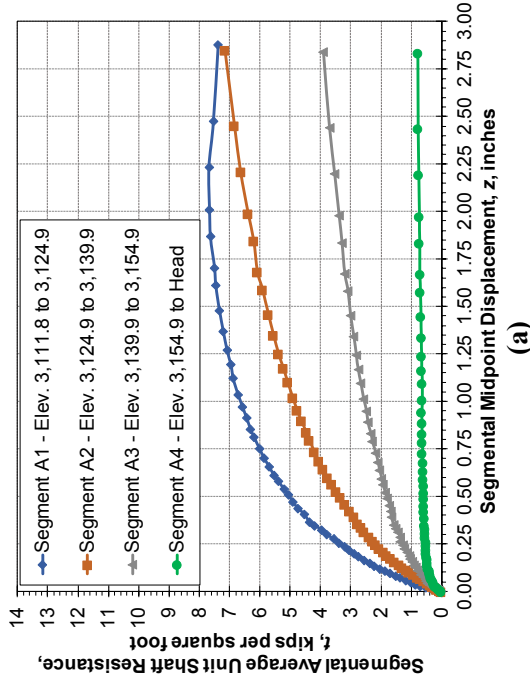
Foundation Displacement-Resistance Relationships – t - z , $Q_{BASE-z_{BASE}}$, $q_{BASE-z_{BASE}}$, and ETL Curves

The factored design load for each of the eight river piers was 2,370 kips (10,540 kN). Applying resistance factors of 0.55/0.50, the required embedment length for each 5.9-foot-diameter (1.8-m) drilled shaft was 66 feet (20 m) below bottom-of-scour elevation (81 feet (25 m) below river bottom to account for 15 feet (4.6 m) of scour). Applying a BDSLT-justified resistance factor of 0.70, required embedment lengths were reduced to 50 feet (15 m) below bottom-of-scour elevation, a length savings of 16 feet (4.9 m) per shaft. This resulted in a total savings of 128 feet (39.0 m) of constructed drilled shaft length. Applying a total BDSLT program cost of \$172,500 to a reduction of \$197,760 in drilled shaft construction cost results in a net savings of \$25,260. The drilled shaft contractor estimates that the reduced shaft lengths saved a total of four days of construction schedule.

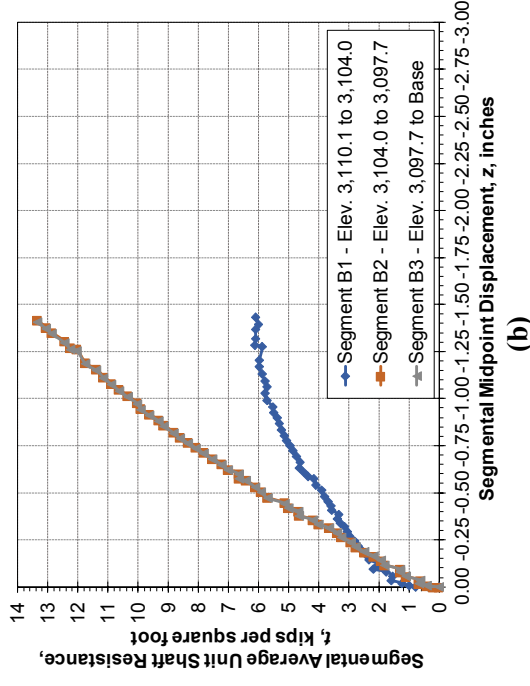
Minimum required embedment lengths to satisfy lateral resistance requirements precluded taking advantage of BDSLT results to additionally shorten the river shafts. For the 18 smaller-diameter abutment and retaining wall shafts, the minimum required embedment length to satisfy lateral resistance requirements controlled design compared to using resistance factors of 0.55/0.50 or 0.70 for resistance to axial demands.

Precluded Remediation

Crosshole sonic logging (“CSL”) testing was performed on each of the northbound structure’s four river shafts. Construction had not commenced on the southbound structure’s four river shafts at the time of this writing; they will also receive CSL testing. CSL testing is performed on almost all large-diameter drilled



(a)



(b)

Fig. 6. Segmental Average Unit Shaft Resistance vs. Segmental Midpoint Displacement, t - z Curves (a) Above Jack Assembly; (b) Below Jack Assembly

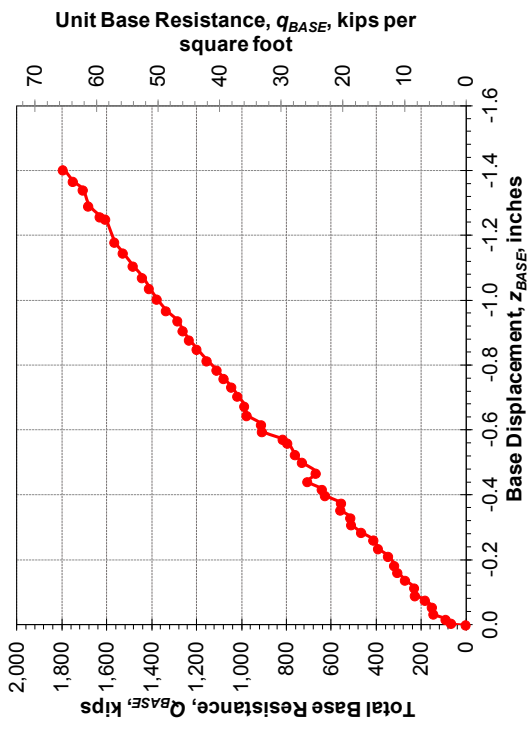


Fig. 7. Base Resistance vs. Base Displacement, Q - z_{BASE} and q - z_{BASE} Curves

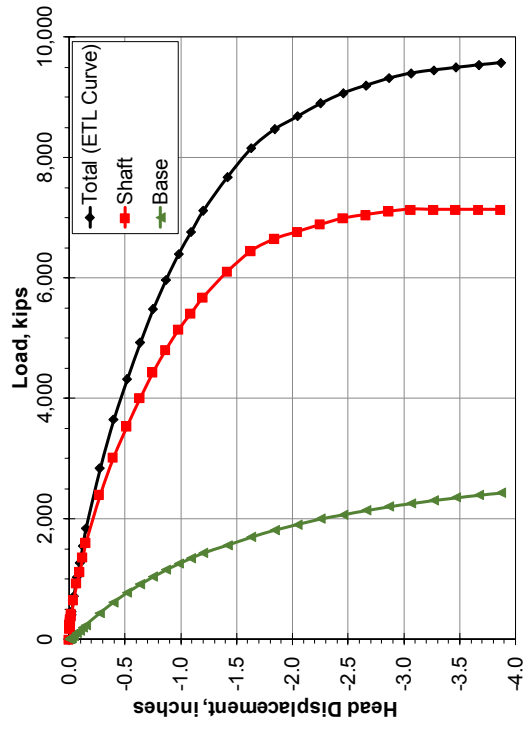


Fig. 8. Equivalent Top-Loading Curve

shafts on MDT projects to evaluate the structural integrity of the shaft concrete. When anomalies are detected, the anomalies are evaluated, and a determination is made as to whether or not the anomaly could affect long-term drilled shaft performance. If the anomaly is determined to be detrimental to shaft performance, the typical procedure is to core the shaft to better-characterize the anomaly. Once coring is performed and the cores and field data are evaluated, a course for remediation is determined. Remediation could include drilling a second core hole and grouting voids within the shaft, grouting or pressure-grouting poor soil zones at the bottom of the shaft through the CSL tubes, or constructing a new adjacent drilled shaft or shafts.

CSL results from the northbound structure's four river shafts indicated varying degrees of anomalies at the base of three shafts, characterized as the potential for weak or soft concrete. Following receipt of the CSL results from the first shaft with an indicated anomaly, the extent of the possible anomaly and its potential impact on the shaft's available base resistance to support design loads was evaluated. Since the apparent anomaly was at the base, it was decided as a first step to review the bi-directional static load test data to determine if shaft resistance alone (ignoring any base resistance) was sufficient to support design loads.

Based on the equivalent top-loading determinations (Fig. 8), the ultimate shaft resistance indicated by the bi-directional static load test was estimated. If base resistance is ignored, the shaft load curve in Fig. 8 can be taken as the equivalent top-loading curve. A number of approaches were used to estimate the ultimate shaft resistance based on BDSLT results, including the modified Davisson formula (slope-tangent) (Kulhawy and Hirany, 2009), the L2 method (Kulhawy and Hirany, 2009), and the FHWA 5% failure criteria (Brown et al., 2010).

The evaluation determined that the subject shaft (the first shaft for which CSL testing results indicated a potential anomaly) had sufficient factored shaft resistance alone to support the design load of 2,370 kips (10,542 kN), ignoring any contribution from base resistance. Similar analyses performed for the two subsequent river shafts for which CSL testing results indicated a potential anomaly also determined that factored shaft resistance alone was sufficient to support the design loads.

As discussed, drilled shafts that contain anomalies are usually remediated by the MDT, particularly when the anomaly is in the bottom several feet of the shaft where crucial base resistance is developed. For this project, base resistance was counted on in the original design. The remediation strategies can be time-consuming, and depending on the extent of coring and/or grouting, it can sometimes take several weeks or more to develop and approve a plan, and perform the remediation. Because a bi-directional static load test was performed at the beginning of this project, data were available to re-evaluate drilled shaft resistances and avoid having to potentially core and/or grout up to three large-diameter drilled shafts, which could have taken three to four weeks or more of project schedule to complete.

Geotechnical Unit Resistances Comparison

A comparison of unit resistance values was performed among 1) historical (published) results from two other bi-directional static load tests performed on drilled shafts with similar diameters and embedded lengths, subsurface conditions, and construction methods: one in Redding, California (Sorensen, 2010), and one in Thompson Falls, Montana (Murfitt, 2015), 2) values predicted for the gravel deposits by SHAFT 6.0 (used for design of the Russell Street Bridge), and 3) the results of the BDSLT performed for this project.

The SHAFT 6.0 model predicts nominal resistances. To compare similar results, an attempt was made to evaluate shaft and base unit resistances from BDSLT results at each of the three sites using the L2 and FHWA methodologies to estimate nominal unit resistances. For the Redding site, the data did not allow for evaluation using either the L2 or FHWA methodologies; the maximum mobilized unit resistance is reported, which underpredicts nominal unit resistances. For the Thompson Falls site, the L2 values were

based on the equivalent top-loading curve construction, and the base did not displace enough to apply the FHWA failure criteria. For the Missoula site, the L2 and FHWA values were based on the ETL curve construction (Fig. 8). The results of the comparison are summarized in Table 2.

Data from the Missoula test indicates shaft and base nominal unit resistance values on the order of twice the values predicted by SHAFT 6.0. It should be recognized that Table 2 presents generalized comparisons,

Table 2. Geotechnical Unit Resistance Values Comparison

Location / Source	Soil Deposit	Shaft Diameter, feet	Head Deflection at Maximum Mobilized Resistance, inches	Geotechnical Unit Resistance, ksf			
				L2		FHWA	
				Shaft	Base	Shaft	Base
Redding, CA BDSLT ¹	Very dense poorly graded gravel, with sand, cobbles, and boulders	8.2	0.4	5.3	N/A	5.3	N/A
Thompson Falls, MT BDSLT ²	Medium dense poorly graded sand and gravel	8.0	1.3	2.99	35.65	N/A	N/A
SHAFT 6.0 Prediction for Missoula Site	Medium dense to dense poorly graded gravel, with sand	5.9	1.1 (L2); 3.2 (FHWA)	2.52	41.27	2.56	41.97
Missoula, MT BDSLT ³	Medium dense to very dense poorly graded gravel, with sand, cobbles, and boulders	5.9	2.8 (L2); 3.2 (FHWA)	5.08	76.84	5.15	84.16

“ksf” = kips per square foot. 1 ksf = 47.88 kPa. “N/A” = Not Available.

Notes:

1. L2 unit shaft resistance values represent maximum mobilized unit resistances; nominal unit resistances are underpredicted.
2. L2 unit shaft and base resistance values based on ETL curve construction.
3. All unit shaft and base resistance values based on ETL curve construction.

and there are many factors that may contribute to differences in results, including the fractured nature of the deposits, groundwater table location, gradation, angularity, and density of the deposits, shaft instrumentation, test protocol, data evaluation and interpretation, construction procedures, etc.

CONCLUSIONS

Deep-foundation designers have many tools and data available to them, including numerous formulas, computer software, basic and sophisticated laboratory testing, and past experience. Often, several of these resources are used to estimate nominal resistances, to which a resistance (or safety) factor is applied to determine a factored resistance (or allowable load). Load testing provides the designer with an improved estimate of nominal resistances, and thus allows use of a higher resistance (or lower safety) factor. However, load testing can be perceived as expensive to some designers, clients, and agencies, viewed as not providing sufficient return on investment. This case history demonstrates that performing load testing on deep foundations can be money well-spent.

In the case history presented, bi-directional static load testing permitted the use of a higher resistance factor than would have been used in the absence of testing. This resulted in saving 16 feet (4.9 m) of embedded length on each of eight river shafts for a new bridge structure. After accounting for the total cost of testing, a net savings of \$25,260 in construction cost was realized, demonstrating that load testing can save more money than it costs (more savings would likely have been realized had the minimum required embedment length to satisfy lateral resistance requirements not controlled designs). Additionally, it is estimated that the decreased required footage saved four days of construction schedule.

More-difficult to quantify in terms of cost and time, but potentially even more-valuable, were the benefits realized from the bi-directional static load test results precluding remediation of three production shafts. When CSL testing indicated potential anomalies near their bases, BDSLT results were used to demonstrate that factored shaft resistance alone (ignoring base resistance) was sufficient to support design loads in three shafts, thereby precluding the need to core or remediate.

An additional benefit from the bi-directional static load testing is the resulting unit shaft and base nominal resistance values indicated by the test. These values can be included in a database of testing results for drilled shafts with similar diameters, soil deposits, and construction procedures. Used with prudence and judgment, such results may allow increased presumptive values to be used for future designs.

REFERENCES

- American Association of State Highway and Transportation Officials, Washington, D.C. (AASHTO) (2011). Guide Specifications for LRFD Seismic Bridge Design, Second Edition, with 2012, 2014, and 2015 Interim Revisions.
- American Association of State Highway and Transportation Officials, Washington, D.C. (AASHTO) (2014). AASHTO LRFD Bridge Design Specifications, US Customary Units, Seventh Edition, with 2015 Interim Revisions.
- Brown, D.A., Turner, J.P., and Castelli, R.J. (2010). Drilled Shafts: Construction Procedures and LRFD Design Methods, FHWA-NHI-10-016, Geotechnical Engineering Circular (GEC) No. 10. U.S. Dept. of Transportation, Federal Highway Administration (FHWA).
- Davisson, M.T. (1972). High Capacity Piles. Proceedings, Soil Mechanics Lecture Series on Innovations in Foundation Construction, American Society of Civil Engineers, ASCE, Illinois Section, Chicago, IL, pp. 81-112.
- Ensoft, Inc. (2007). SHAFT – A Program for the Study of Drilled Shafts Under Axial Loading, Version 6.0 for Windows.
- Kulhawy, F.H., and Hirany, A. (2009). Interpreted Failure Load for Drilled Shafts via Davisson and L1-L2. Geotechnical Special Publication. 127-134. 10.1061/41021(335)16.
- Murfitt, T. (2015). Osterberg Cell Test Report, Thompson River – East. STPB-STPP-HSIP 6-1(106)56, Contract: 12514, Experimental Project MT-13-02.
- Rollins, K.J., Clayton, R., Mikesell, R., and Blaise, B.C. (2005). Drilled Shaft Side Friction in Gravelly Soils. Journal of Geotechnical and Geoenvironmental Engineering – J GEOTECH GEOENVIRON ENG. 131. 10.1061/(ASCE)1090-0241(2005)131:8(987).
- Sorensen, K. (2010). Effects of Gravels, Cobbles, and Boulders on Capacity of Large Diameter Shafts at the Cypress Avenue Bridge, Redding, California. Proceedings, Art of Foundation Engineering Practice Congress, 2010.